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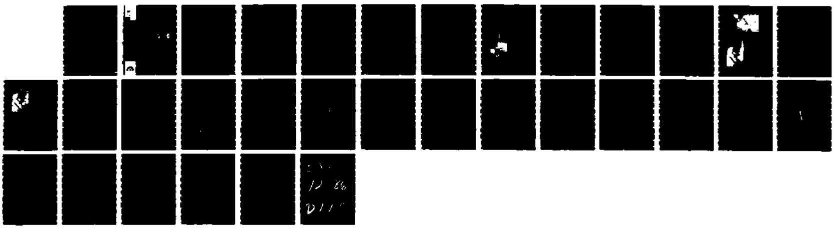

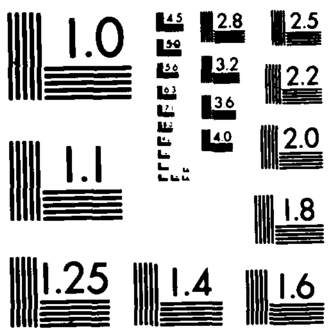

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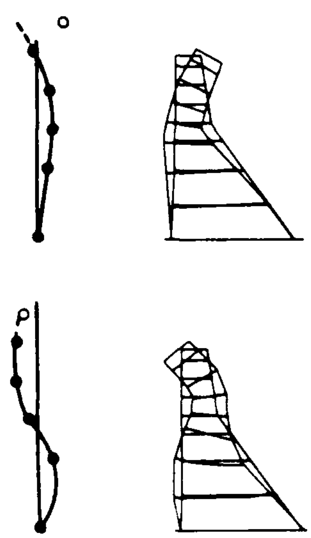

TECHNICAL REPORT SL-86-33

DYNAMIC FAILURE TESTS AND ANALYSIS OF A MODEL CONCRETE DAM

by

C. Dean Norman

Structures Laboratory

DEPARTMENT OF THE ARMY
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US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

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A 1:60-scale model of the highest nonoverflow monolith of Koyna Dam was constructed on a foundation block and subjected to an impact force which produced cracking in the upper portions of the dam. The foundation block was connected to a massive concrete reaction structure by Belleville springs. The impact force was provided by a 1,000 lb wrecking ball mass moving as a pendulum. Instrumentation consisted of embedded strain gages and accelerometers attached to the foundation block and the upstream face of the dam. Vibration tests were conducted prior to the impact test and afterwards to determine natural frequencies and mode shapes. The initial velocity of the dam foundation system during the impact test was estimated at 25 inches per second. Internal strain time histories are presented and discussed. Damage to the model dam is compared to damage occurring in Koyna Dam during the Indian earthquake of 1967.

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PREFACE

This study was conducted during the period October 1979-January 1981 by personnel of the US Army Engineer Waterways Experiment Station (WES) under sponsorship of the Office, Chief of Engineers, US Army. This report was written by Mr. C. Dean Norman, Concrete Technology Division, Structures Laboratory (SL). Mr. Harry Stone and Dr. Paul Mlakar, SL, conducted the tests and collected the data.

This study was conducted under the supervision of Mr. James T. Ballard, former Chief, Structural Mechanics Division, current Assistant Chief, SL. Mr. Bryant Mather was Chief, SL.

COL Allen F. Grum, USA, was the previous Director of WES. COL Dwayne G. Lee, CE, is the present Commander and Director. Dr. Robert W. Whalin is Technical Director.

CONTENTS

DYNAMIC FAILURE TESTS AND ANALYSIS OF A MODEL CONCRETE DAM

PART I: INTRODUCTION

Background

1. In assessing the design of a new dam or the seismic safety of an existing dam, the maximum design earthquake is usually the controlling maximum credible earthquake. The maximum credible earthquake is defined as the largest earthquake associated with a specific seismotectonic structure or source area within the region examined [1].* For the maximum credible earthquake loading, inelastic structural behavior with associate cracking of the concrete in a gravity dam is permissible as long as the cracking is not severe enough to cause a significant failure of the dam (resulting in release of the reservoir). Since current construction costs for dams of moderate size and reservoir capacity are upward of $200 million and since the occurrence of a failure would result in immeasurable costs in life and property damage, results from earthquake structural analyses are evaluated very carefully by licensing agencies and their consultants before a particular design or safety evaluation is approved. Many existing dams and future dam sites are located in areas where design ground accelerations are in the range of 0.25 to 0.5 g's. Consequently, state-of-the-art earthquake structural analyses will generally indicate tensile stresses in many of these dams from 150 to 650 psi [2 and 3]. Not only is the validity of these "tensile stresses" (predicted generally from linear analyses) questioned, but also there is no way to confidently predict the overall level of damage to a concrete dam subjected to this state of stress based on a linear analysis.

2. In light of the problem areas discussed above, a dynamic failure test of a model concrete dam was conducted to provide quantitative failure test data. The intent was to study the response characteristics of a model dam subjected to a simple well defined horizontal component of motion at the base of the model.

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* Numbers in brackets refer to references at the end of the report.

Objective

3. The objective of this study was to conduct and evaluate a dynamic failure test of a model concrete dam.

Scope

4. A 1:60-scale model of the highest nonoverflow monolith of Koyna Dam [4] was constructed on a foundation block as shown in Figure 1. The foundation block was connected at one end to a massive concrete reaction structure through a series of Belleville springs. Input motion to the foundation block was provided by impacting the other end with a steel mass. Strains in the model dam and accelerations on the dam surface were monitored.

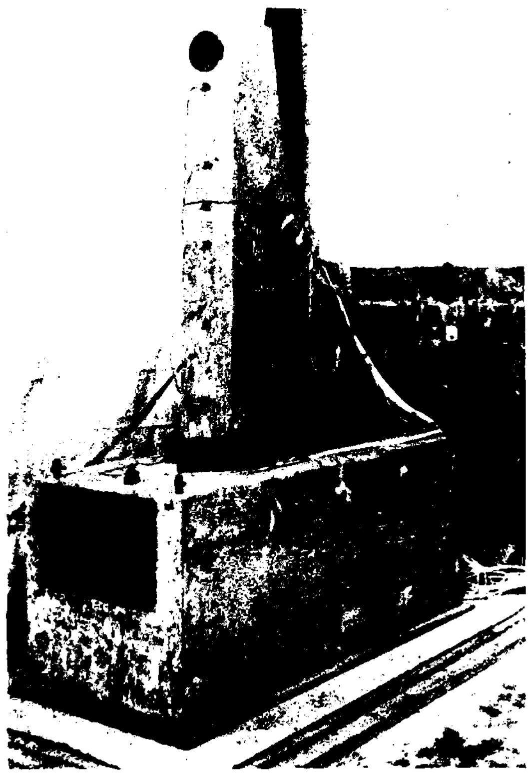


Figure 1. 1:60-scale model and foundation block of tallest nonoverflow section of Koyna Dam

PART II: KOYNA DAM, PROTOTYPE AND MODEL CHARACTERISTICS

Prototype Dam

5. As discussed in detail in [4], Koyna Dam is a straight gravity structure made of rubble concrete consisting of 9-in.-thick layers of plastic air-entrained matrix concrete with 4- to 16-in. size rubble placed on top of each layer. The concrete and rubble layers were vibrated and then the surface was prepared using procedures similar to those adopted for conventional concrete before the next layer of plastic concrete was placed. To provide a more impervious zone, conventional concrete with maximum aggregate size of 3 in. was provided for 6-ft thickness at the upstream face, of all monoliths, and also at the downstream face of the overflow monoliths. The last six monoliths near the left bank are not made of rubble concrete, but of hand-laid rubble masonry.

6. The general plan, elevation, and sections of Koyna Dam are shown in Figure 2. The dam is about 2,800 ft long, 280 ft high above the river bed, and 338 ft high above the deepest foundation. The dam is constructed in 50-ft-wide monoliths, and the contraction joints between the monoliths are provided with copper water seals. The spillway or overflow part of the dam is about 300 ft long.

7. Compressive and tensile strengths for three of the four different mixes used in Koyna Dam are listed in the table below (from [5]); only a small volume of mix No. 1 was used in the extreme lower parts of the dam and is therefore excluded from the table.

Mix No.	Parts of the Dam	Compressive Strength (psi)	Assumed Tensile Strength (psi)
2	Up to KRL* 1900	4100	410
3	KRL 1900-2160	3500	350
4	Above KRL 2160	2900	290

* Elevations are given in terms of Koyna Reduced Level (KRL) whose datum is above mean sea level by 31.0 ft.

Model Dam

8. Dimensions of the 1:60-scale model dam and a photograph of the dam and foundation are presented in Figures 3 and 1, respectively. The

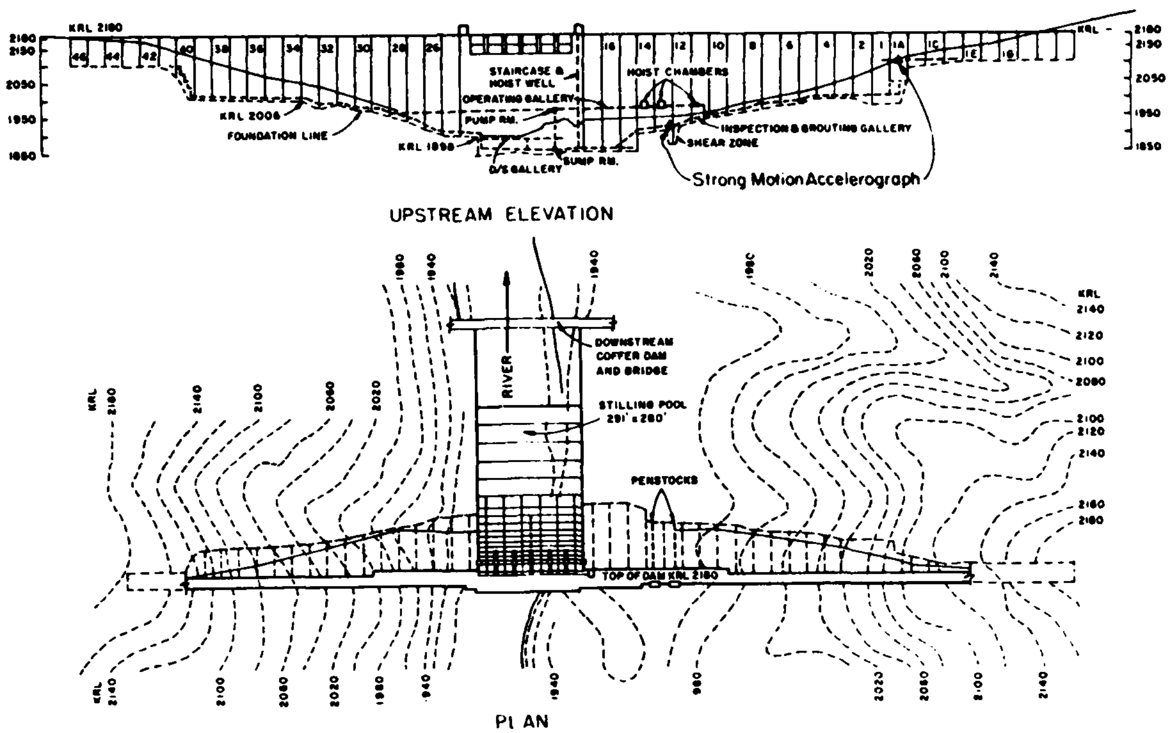


Figure 2. Koyna Dam prototype, plan and elevation. From [3]

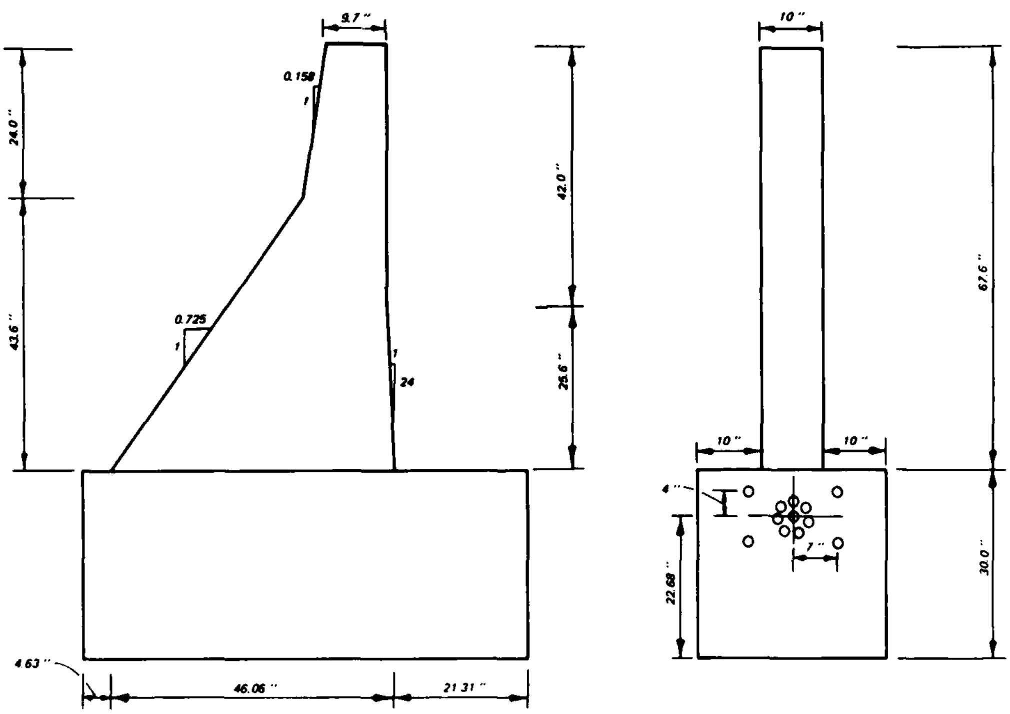


Figure 3. Dimensions of the 1:60-scale model of Koyna Dam

10-in.-wide model monolith simulating the tallest nonoverflow monolith of Koyna Dam was cast with only one concrete mix. The results from five unconfined compression tests of the model concrete mix are presented below.

Concrete Property	Mean Value	Coefficient of Variation
f'_c (psi)	2040	4.839
Secant* Modulus (psi)	3.34×10^6	1.807
Poisson's* Ratio	0.237	13.221

* Values obtained at a stress level of 0.5 f'_c.

A steel plate 1 in. thick and 6.5 in. in diameter was epoxied on the upstream face of the model near the top. The plate provided the connection for a small vibrator used to determine natural frequencies and mode shapes for the model dam. The model dam section was cast monolithically with the foundation.

9. The foundation block is 30 in. by 30 in. by 72 in. long with four 1-in. steel bars extending the length of the block and connecting steel base plates for the Belleville springs on one end and the metal impact mass on the other. The Belleville spring system is shown in Figure 4, and the model and impact mass are shown in Figure 5. The Belleville spring system provides a simple way to increase or decrease the period of the base motion (e.g., as more Belleville springs are added the period decreases). Also the response of the individual springs was restricted to remain linear so that the total spring system response would be linear and several oscillations of the dam-foundation system could be obtained. The foundation block rests on a smooth plane surface prepared on the test floor using a grout mixture. A layer of Teflon was then placed over the plane grout surface. The base of the foundation block was also covered with a Teflon layer to minimize frictional resistance to the motion of the model dam-foundation system. The impact mass is a 1,000-lb steel sphere suspended by a long flexible steel cable.

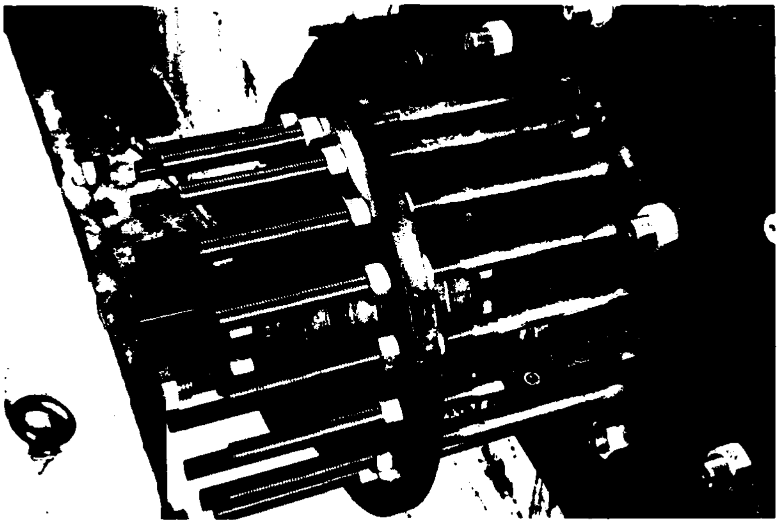


Figure 4. Belleville Spring system connecting model dam foundation block and massive concrete reaction structure

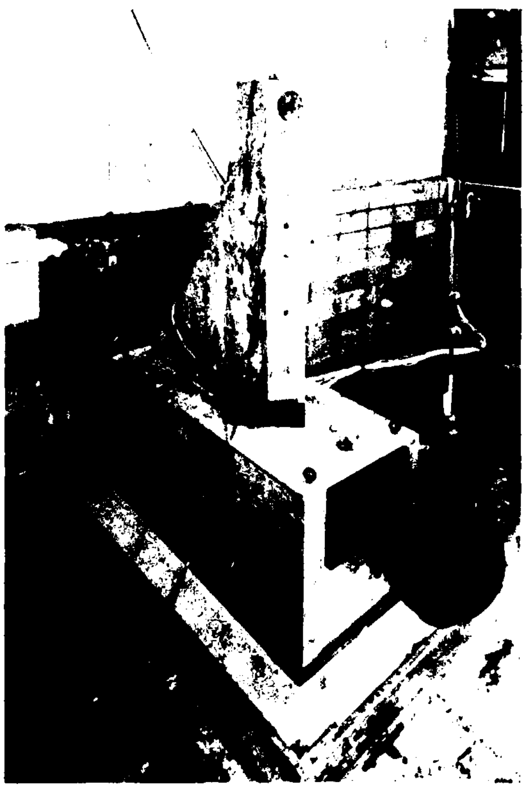


Figure 5. Model dam and 1,000 lb impact mass

PART III: TESTS CONDUCTED - INSTRUMENTATION

Vibration Test

10. Prior to the dynamic failure test, forced vibration tests were conducted on the model dam to experimentally determine natural frequencies and mode shapes. One electromagnetic (EM) vibrator was attached to the circular steel plate which was epoxied to the upstream face of the dam near the crest. During the vibration tests the frequency of the vibrator force was continuously varied over a range of 100 to 1,000 Hz, with a constant maximum force amplitude of 20 lb. After trying several frequency sweep rates, a value of 0.25 decades per minute was selected to provide quasi-steady-state results. The model with vibrator in place is shown in Figure 6. Vibrator control was provided by a Spectral Dynamics Servo-Monitor to maintain constant maximum force amplitude and a Spectral Dynamics sweep oscillator for maintaining a constant sweep rate. The maximum force capability of the vibrator is 50 lb, and the maximum frequency range is 15 to 5,000 Hz. The vibrator was driven by a 150-watt power amplifier.

11. Eight piezoelectric accelerometers were distributed along the upstream face of the model and on the foundation block for the vibration tests. The exact locations of these accelerometers are shown in Figure 7. Accelerometers mounted on the foundation block and on the crest of the model dam recorded lateral and vertical accelerations in addition to motion in the direction of the long axis of the foundation block. These transducers were interfaced by compatible electronics to an analog magnetic tape recorder.

Dynamic Failure Test

12. The dynamic failure test was designed to subject the model dam to a severe horizontal component of base motion of a periodic form. This was achieved by connecting the foundation block to a very massive concrete reaction structure through a system of Belleville springs. The spring system could be modified to produce different frequencies of base motion. The input force was developed by impacting a 1,000-lb steel sphere suspended as a pendulum and striking the foundation at the center of mass of the model dam-foundation system. Preliminary single-degree-of-freedom analyses were

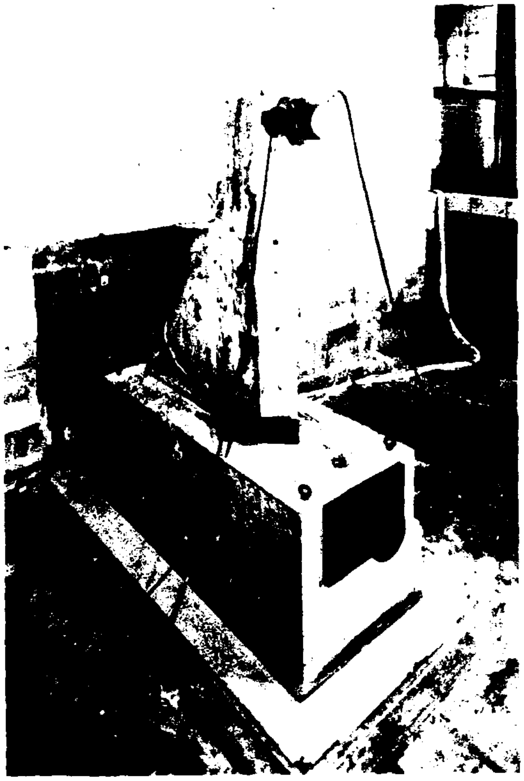
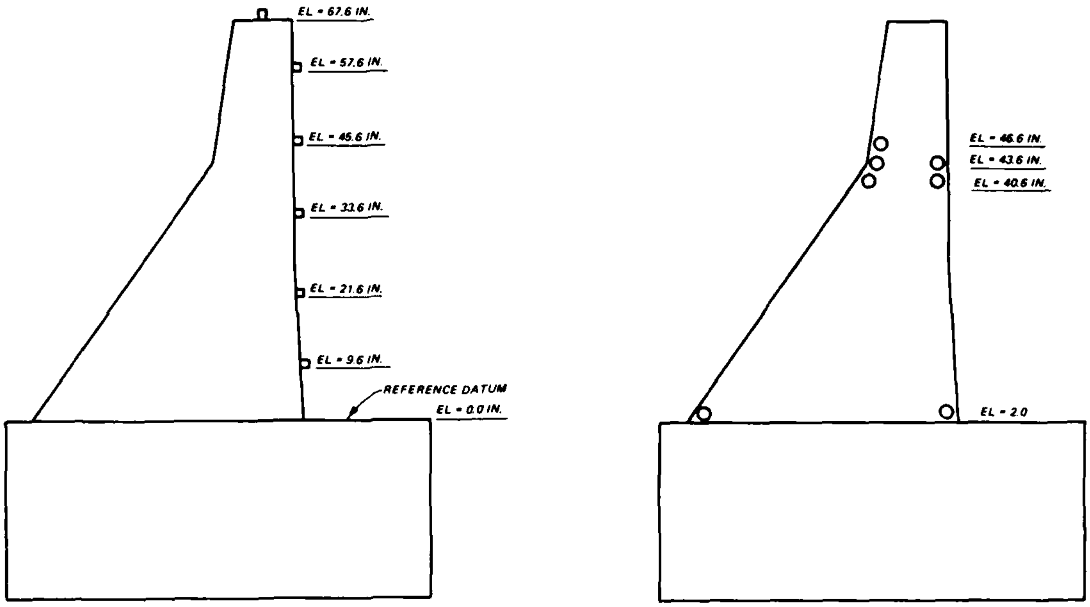


Figure 6. Model dam with EM vibrator in place



a. Location of accelerometers

b. Location of embedded strain gages

Figure 7. Instrumentation plan

conducted to determine spring frequencies and mass drop heights which would produce failure stress levels in critical regions of the model dam. The spring system selected consisted of six Belleville springs confined and preloaded by steel plates which enabled the system to respond in both tension and compression.

13. As in the case of the vibration tests, accelerometers were also used in the dynamic failure test. Accelerometers on the foundation block and on the crest of the dam recorded accelerations in three mutually orthogonal directions as was done in the vibration tests. Only one accelerometer was monitored on the upstream face of the dam at an elevation of 39.6 in. above the top of the foundation block. In addition to the accelerometers, 2-in. embedment-type strain gages were placed at critical regions throughout the model dam cross section as shown in Figure 7. Similar electronics to that used in the vibration test were used in the dynamic failure test.

PART IV: TEST RESULTS

Vibration Tests

14. Vibration tests data were analyzed using the Coincidence-Quandrature (CO-QUAD) method which is based on decomposing structural motions into real and imaginary components [6]. Natural frequencies for the model section are presented in the table below along with scaled values of prototype frequencies based on the finite element analysis reported in [4]. In the table two values for the third mode, model (uncracked), natural frequency are presented because each of these frequencies indicated resonance characteristics associated with the mode shape of the third mode.

	Mode 1	Mode 2	Mode 3	Mode 4
Prototype*	184	492	645	952
Model (uncracked)	206	453	604 702	900
Model (cracked)	328	529	627	817

* Prototype frequencies from finite element analysis; frequencies in hertz.

15. Mode shapes from the finite element analysis of [4] are compared with shapes determined from the model vibration tests in Figure 8. Mode 3 which is in general a vertical response mode could not be precisely reproduced in the model tests since most of the accelerometers were oriented to measure horizontal motions. For the model tests mode shapes the points represented by open circles are generally inconsistent with the other points measured in a particular test.

Failure Test

16. Unfiltered strain versus time plots from the embedded strain gages used in the dynamic failure test are presented in Figures 9 through 17. Zero time for each of these plots should be taken at the time when the first relatively large spike occurs on the record. At each point (Figure 7) three strains are recorded and presented in each figure. Also, strains were measured on short No. 2 vertically oriented rebars placed near the heel and toe at the dam-foundation interface. These rebars were placed at the

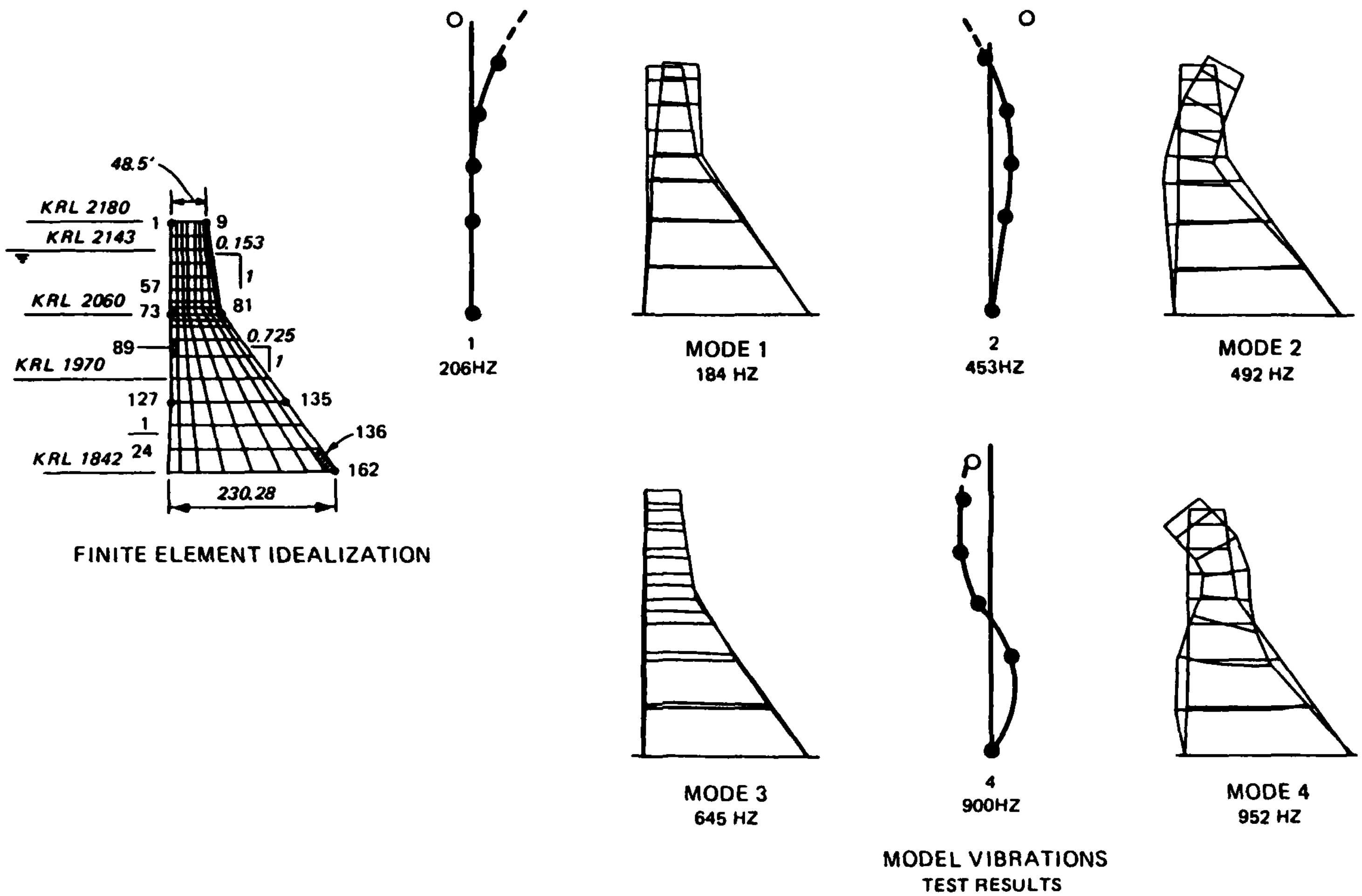


Figure 8. Natural frequencies and mode shapes from finite element analysis [4] and from model tests

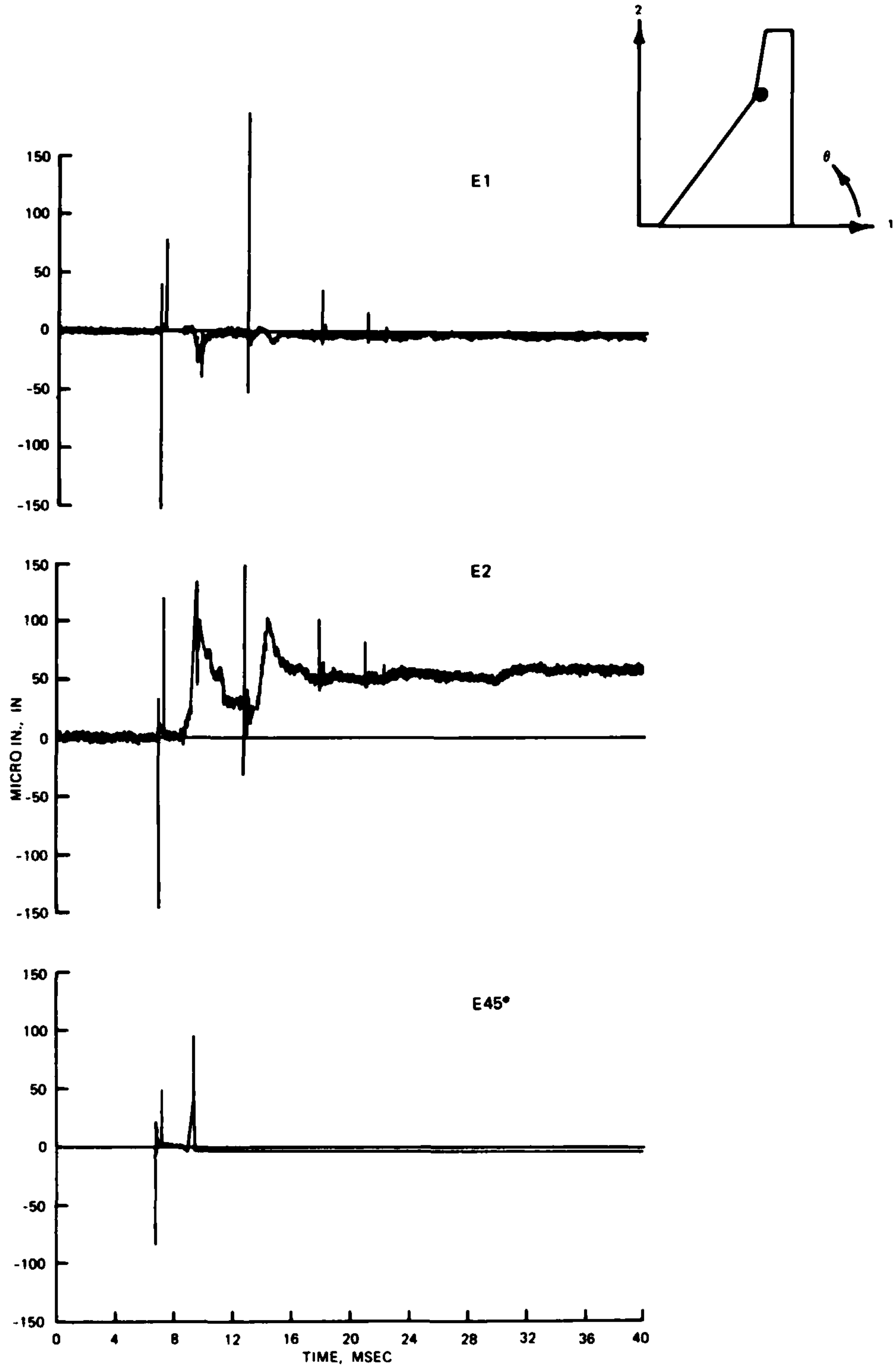


Figure 9. Strain versus time, downstream face at elevation 46.6 inches

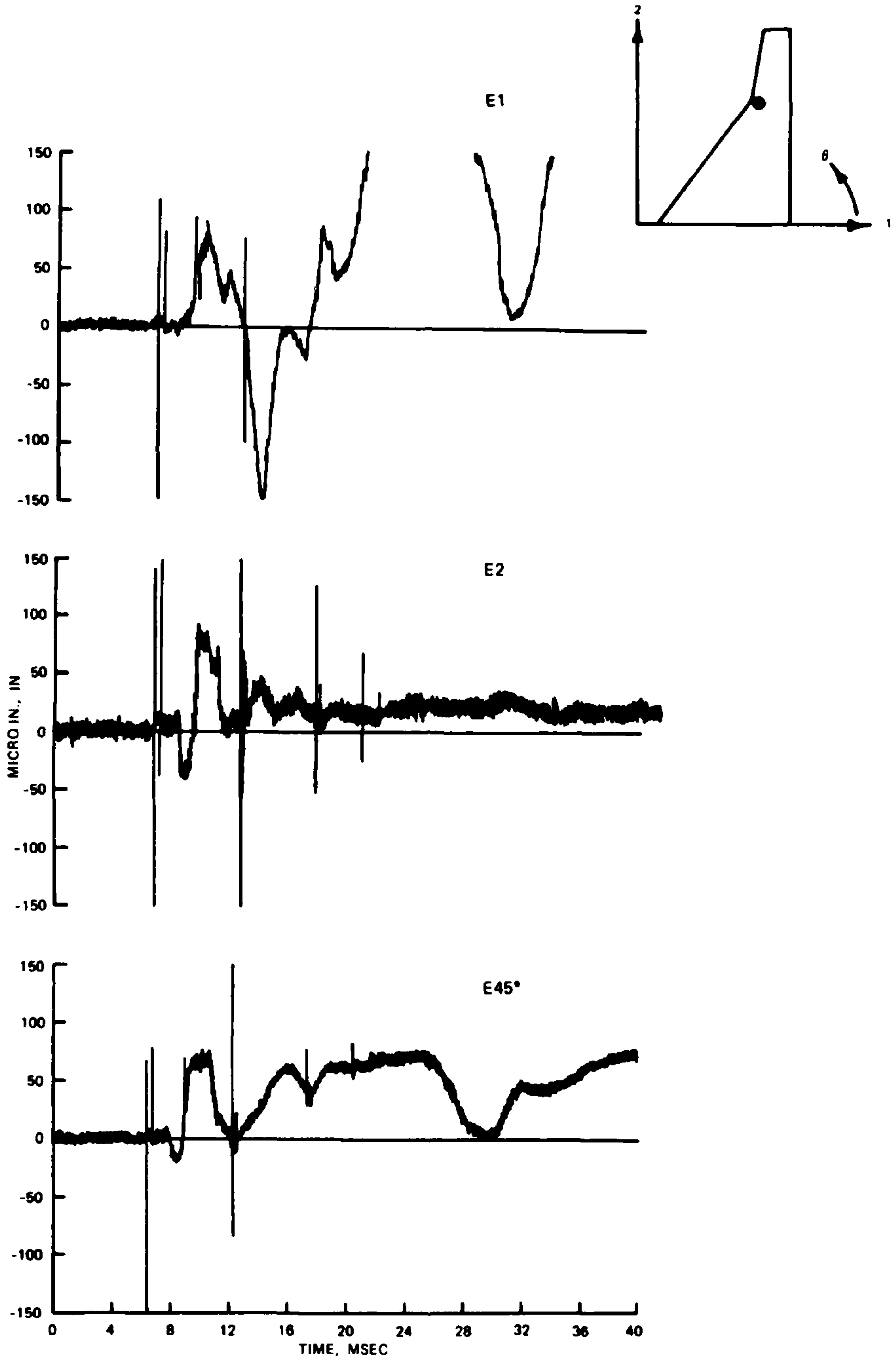


Figure 10. Strain versus time, downstream face at elevation 43.6 inches

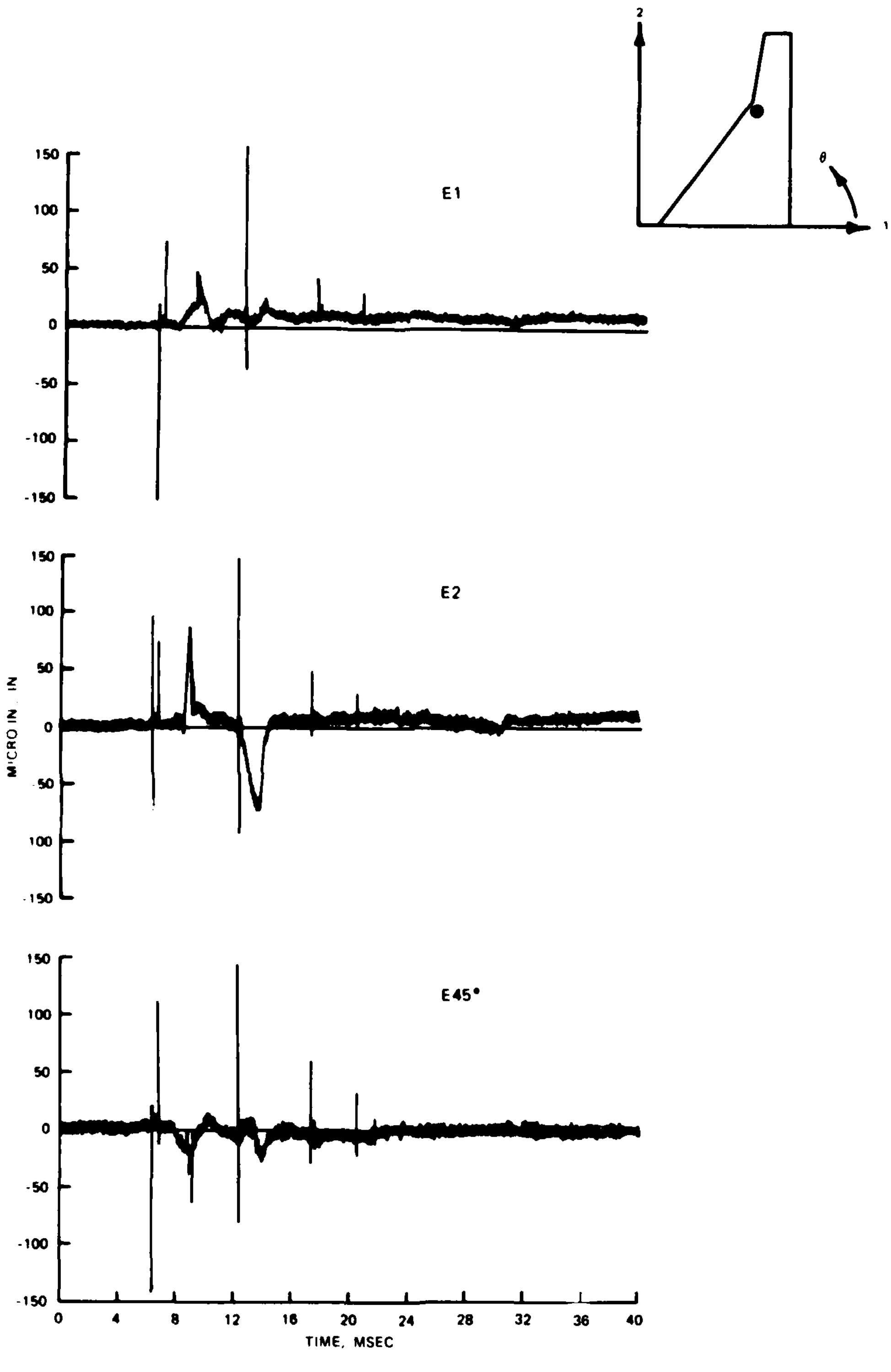


Figure 11. Strain versus time, downstream face at elevation 40.6 inches

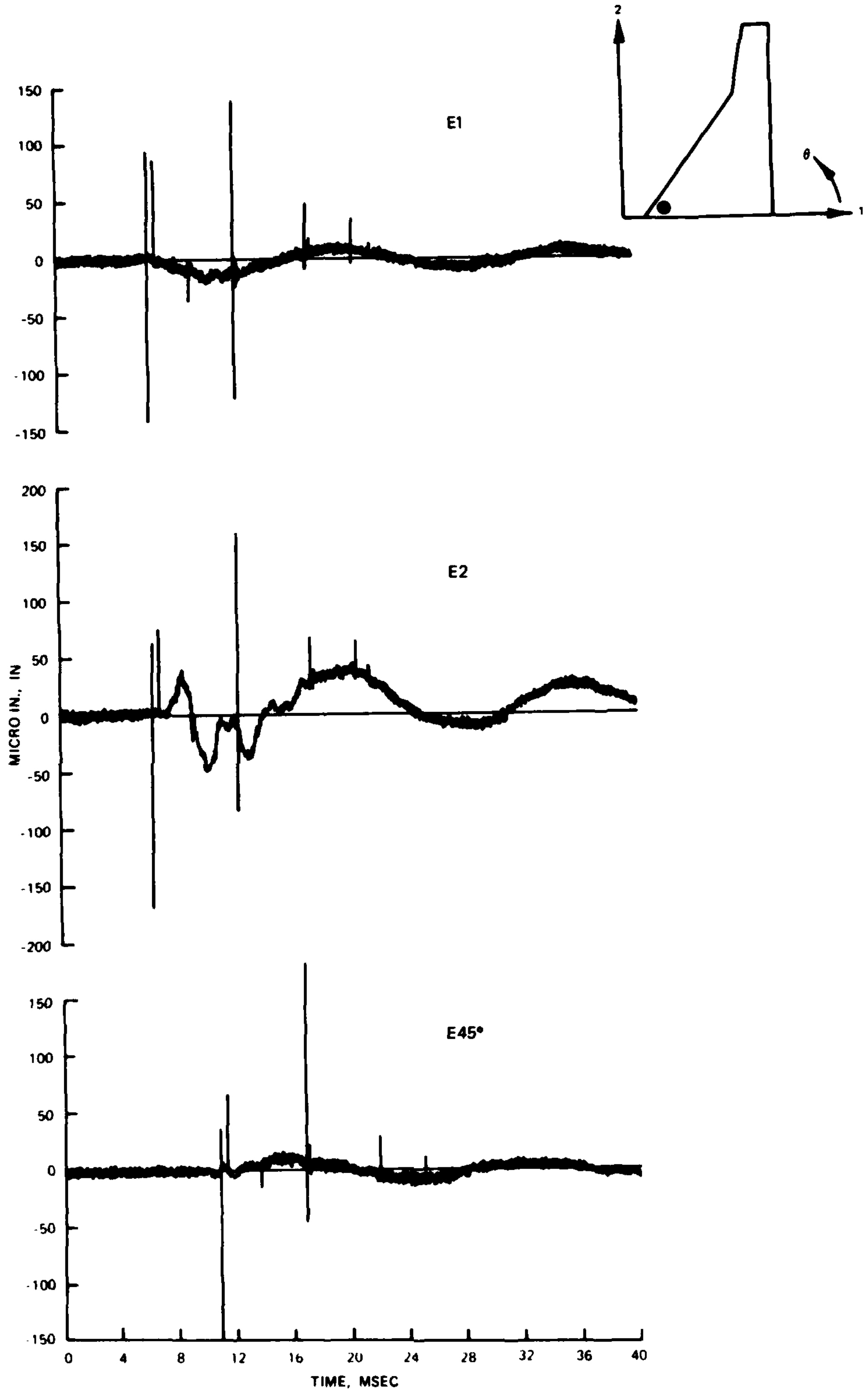


Figure 12. Strain versus time, downstream face at elevation 2.0 inches

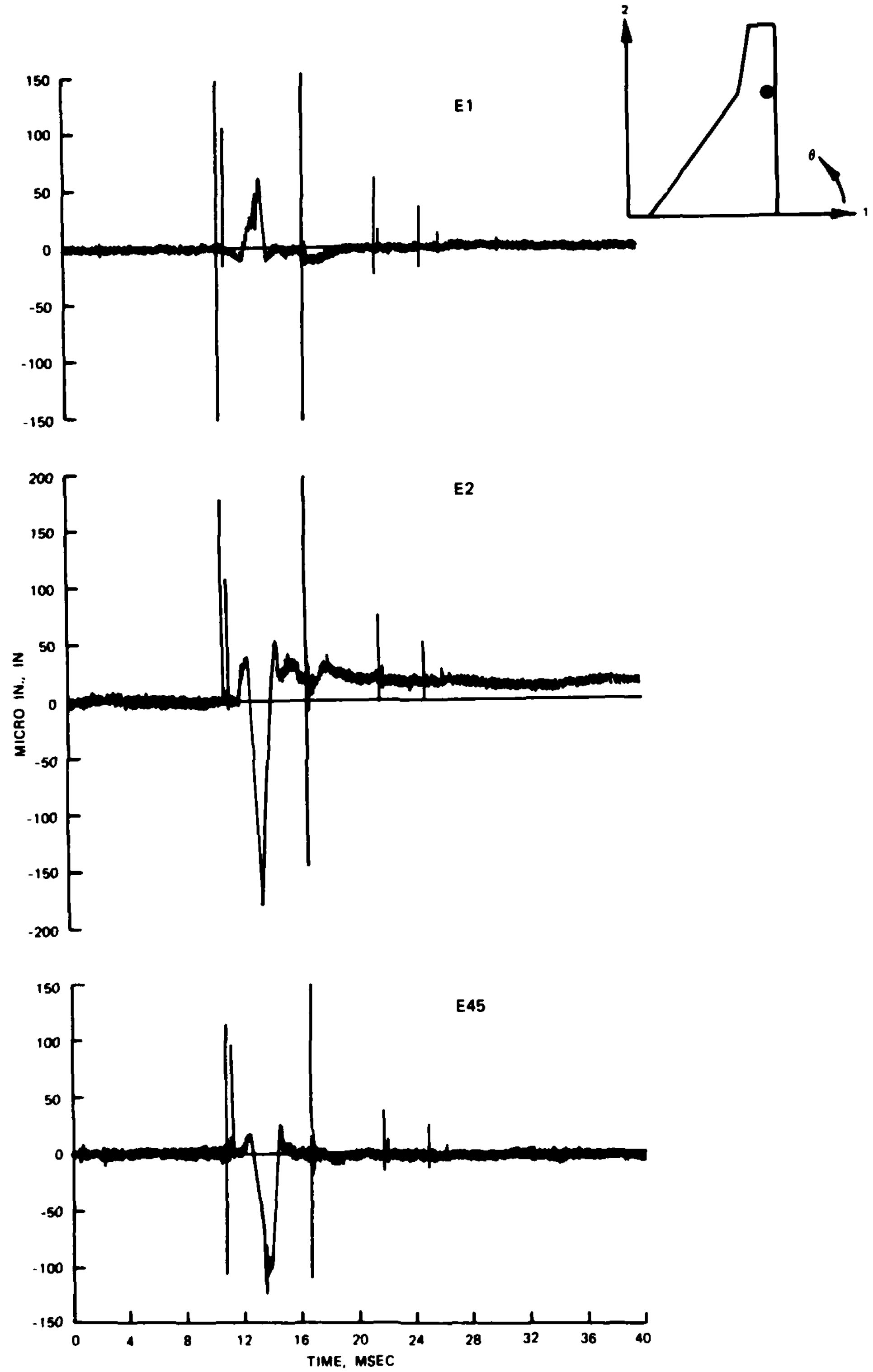


Figure 13. Strain versus time, upstream face at elevation 43.6 inches

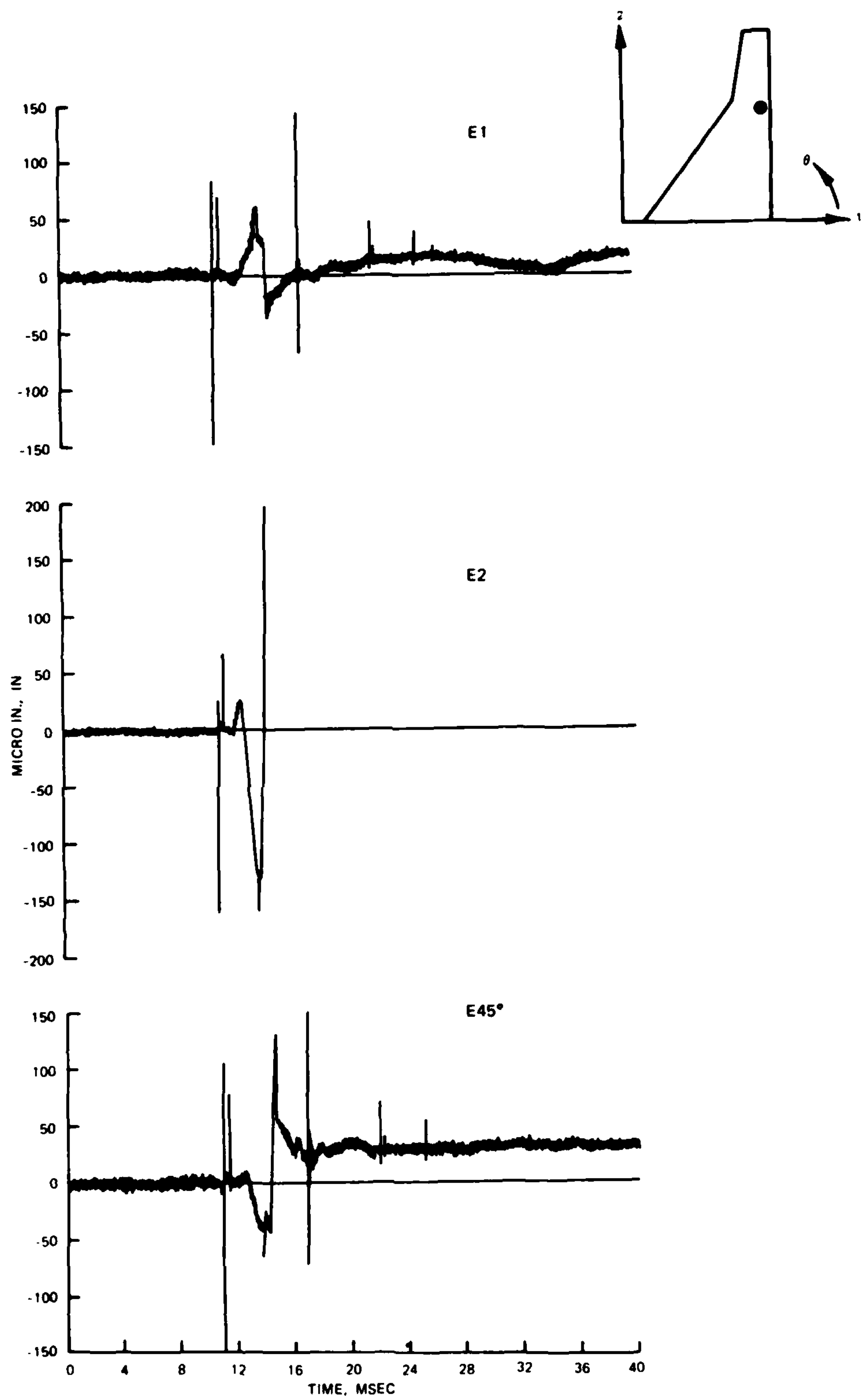


Figure 14. Strain versus time, upstream face at elevation 40.6 inches

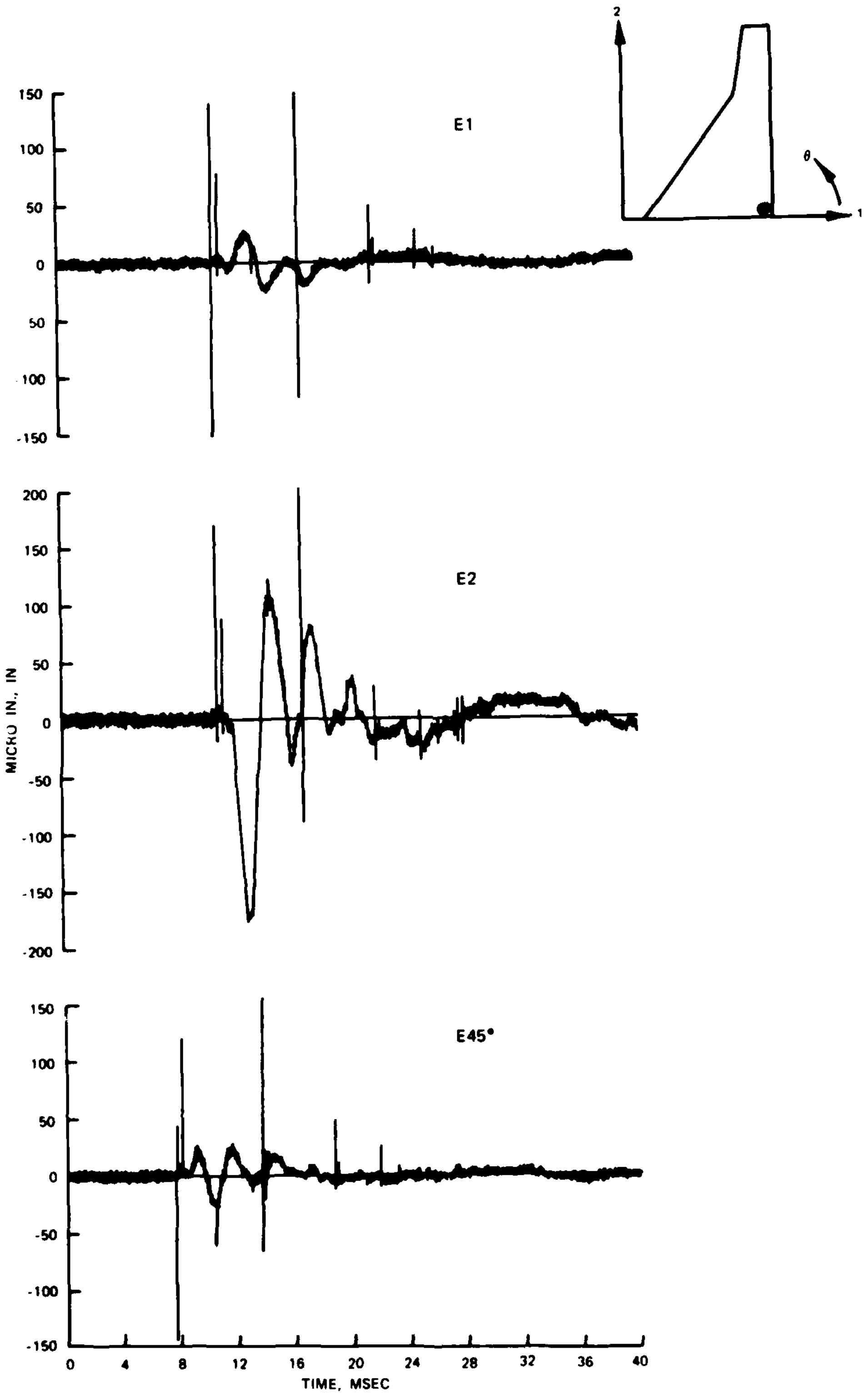


Figure 15. Strain versus time, upstream face at elevation 2.0 inches

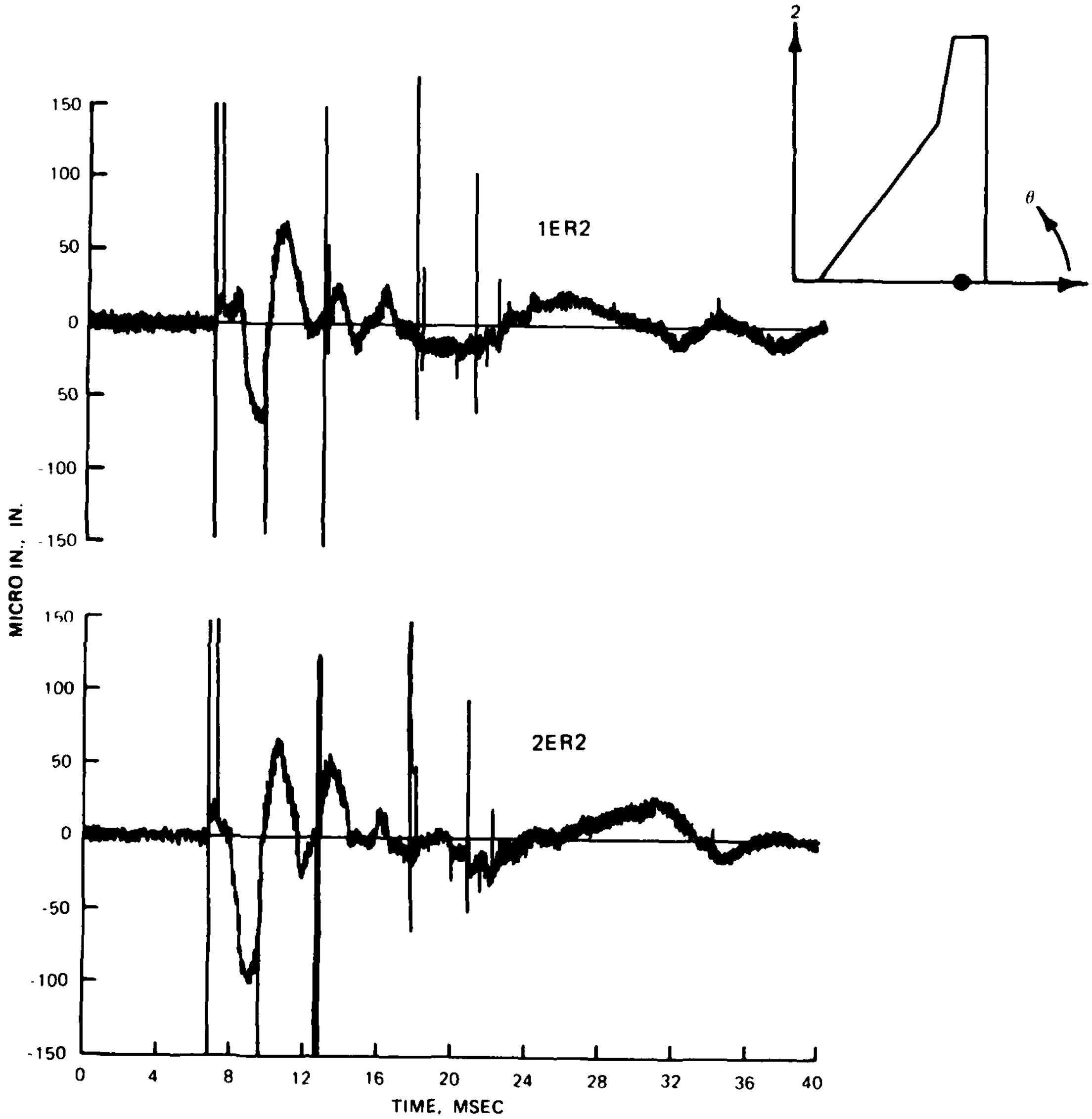


Figure 16. Strain versus time, near upstream face, at elevation 0.0 inches, on rebar

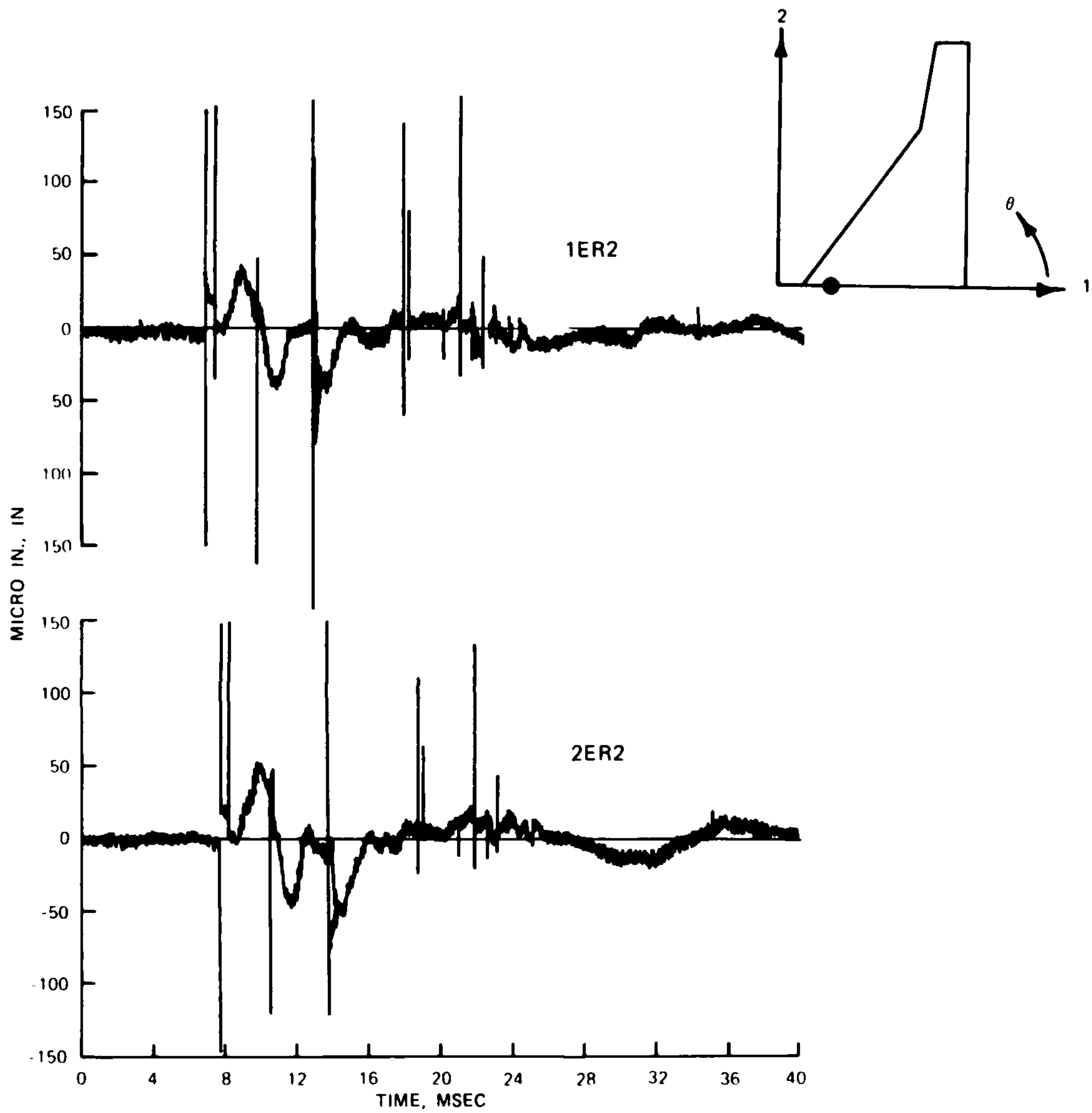


Figure 17. Strain versus time, near downstream face, at elevation 0.0 inches, on rebar

dam-foundation interface to insure there would be no shear or sliding type failures at this level due to the fact that gravity is not scaled in the modeling procedure used.

17. As mentioned previously, all of the accelerometers were clipped in the failure test. However, the accelerometer at approximately 39 in. above the top of the foundation block (Figure 1) appeared to record reasonable accelerations up to the time of maximum velocity. This can be seen in Figure 18. In this figure obvious clipping of the record occurred at about 11 ms. The peak velocity of approximately 25 in. per sec is close to that predicted by impact calculations made in Part V of this report.

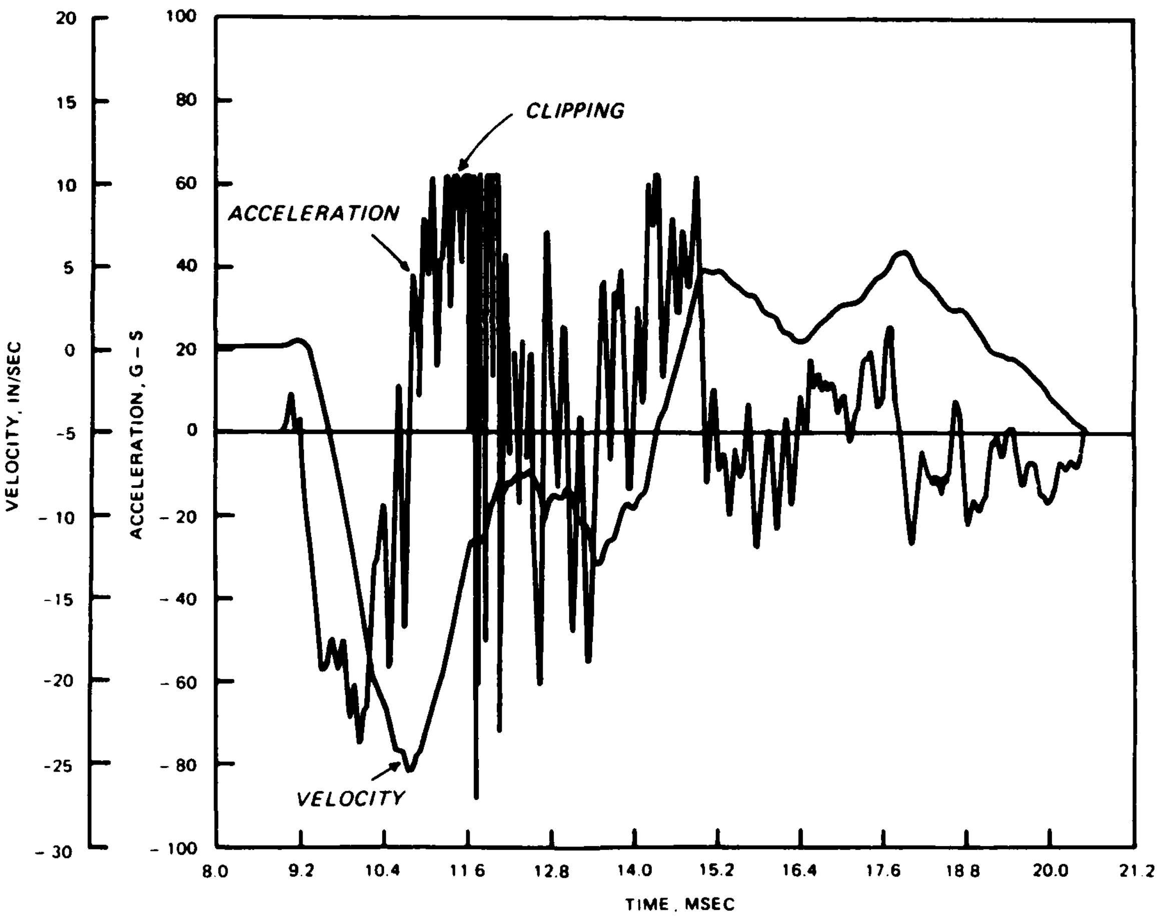


Figure 18. Acceleration and velocity versus time, measured on the upstream face, at approximately thirty nine inches above the foundation block

PART V: ANALYSIS AND DISCUSSION OF TEST RESULTS

Vibration Tests

18. Natural frequencies and mode shapes measured for the model compare quite well with similar scaled predictions for the prototype as presented in Figure 8. Mode 3, an essentially uniform vertical extension mode, could not be geometrically reproduced by model test results since gages on the face of the dam measured motions normal to the face. However, a resonant peak at 604 Hz was recorded by the gage on the crest which measured vertical motions. The cracked model frequencies were obtained from vibration tests after the failure tests. Although a surface crack was observed completely around the perimeter of the model (Figure 1) after conducting the failure test, the crack did not extend throughout the interior of the cross section. This conclusion was based on the fact that a moderate horizontal force was applied to the section above the crack to see if that section was still stable, and there was no movement of the section. Also, the EM vibrator was again affixed at the crest and postcracking vibration tests were conducted with no observation of crack growth or instability of the top section during these tests. In addition to the postcracking vibration tests a second impact test was conducted. In this test there was no indication of crack growth or instability.

Failure Test

19. As mentioned earlier, the accelerometer at the model foundation level was to be used to define the input base motion for analysis and evaluation of structural response. However, since all accelerometers were ranged too low, gage signals saturated or clipped as shown in Figure 18. However, the gage represented by Figure 18 (at an elevation of 39 in. above the foundation) can be used as an indication of the initial acceleration and velocity of the dam foundation system. The initial peak in velocity occurs prior to the major clipping of the gage and is approximately $\nu_i = 25$ in./sec. If the dam is considered to be stiff, then the accelerations at about midheight can be assumed to be about equal to those at the foundation. The input motion at the foundation might then be characterized as a velocity pulse with an amplitude of 25 in./sec and a rise time similar to the early time portion of the signal

shown in Figure 18. An estimate of the input motion can also be made based on the results of a simple impact problem solution. If a coefficient of restitution of approximately 0.85 is assumed (e = 1 for purely elastic impact) for the pendulum mass which had a drop height of 21 in. and a weight of 1,000 lb, the velocity of the dam-foundation system can be estimated at approximately 25 in./sec.

20. As mentioned earlier, measured strains versus time records are presented in Figures 9 through 17. Also, zero time for each record should be taken as that time when the first spike occurs on that particular record. The crack on the upstream face occurred at an elevation of approximately 39 in., which is very near the elevation of the strain gage whose record is presented in Figure 14. Also, the crack on the downstream face occurred close to gages whose records are presented in Figures 9 and 10. Based on these records, it appears the crack formed somewhere between 2.5 and 3.0 ms after impact. Strain measurements at the base near the heel are presented in Figure 15 and near the toe are presented in Figure 12. The vertical strains oscillate at a frequency of about 250 Hz for 8 to 9 ms, then a slow oscillation at about 60 Hz is observed in Figures 12 and 15. Strains near the heel and toe occurring at the slower frequency are felt to be the results of the dam responding as a rigid body rocking on the foundation which is oscillating due to the Belleville springs near the 60-Hz region.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

21. Although motion gages were calibrated too low which limited their use for input load definition many of the strain records provided useful information in characterizing the dynamic response of the model concrete dam. Based on this fact and the assumption that accurate calibration of the accelerometers would have led to a good definition of input motion it is concluded that the test concept reported herein can be effectively used to better define failure criteria for concrete dams subjected to dynamic loads.

22. Based on the location of cracking relative to that experienced at Koyna Dam and the frequency of recorded strains near the base of the model it is concluded that the general features of the dynamic response of the model dam reported herein are similar to those which would be predicted for a prototype dam subjected to strong ground motion with predominant components in the horizontal plane and normal to the long axis of the dam. This is not to say that the motion input to the model dam simulated any particular earthquake. However, the model test concept could be used to establish general levels of energy associated with a relatively pure harmonic input which would cause failure in a particular model dam. The test concept could be applied with a variety of input motion frequencies to different classes of dam materials, geometries, and foundation conditions. Also, given design earthquake acceleration time histories or response spectra could be compared and evaluated with model input motion, through harmonic analysis.

Recommendations

23. It is recommended that the dynamic failure test concepts presented in this report be evaluated, and refined. Subsequently a carefully planned expanded test program should be conducted. Using the test program reported herein as a baseline an expanded program should include:

- a. Surface strain gages in addition to embedded gages.
- b. Test of different size models and different strengths of concrete.
- c. Alternative sources for load input such as dynamic actuators.

d. Various combinations of Belleville springs.

e. The use of a calibrating foundation block so that a particular test input motion can be determined prior to the actual test.

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